SECTION 11

DROP SPILLWAYS

1. GENERAL

Description. The drop spillway is a weir structure. Flow passes through the weir opening, drops to an approximately level apron or stilling basin, and then passes into the downstream channel. The basic elements of the drop spillway and the nomenclature are shown by drawing ES-63 (page 1.3). Various designs and proportions are in use. Further research and systematic evaluation of experience with existing structures will lead to continued improvement in design criteria.

Material. For most soil conditions, drop spillways may be built of any of the construction materials adapted for use in hydraulic structures. Some or all of the following materials will be available for consideration in any locality: concrete, reinforced or cyclopean; rock masonry; concrete blocks, with or without reinforcing; and steel sheet piling. Reinforced concrete is most widely used and has been very satisfactory for long-life, low-annual-cost structures. In a given case, particularly where a number of structures are involved, the selection of the material to be used should be based on: (1) the required life span of the structures, (2) an annual cost comparison which recognizes all of the costs, including maintenance and replacement, for structures built of the different available materials.

Functional Use. Drop spillways are used for the following purposes:

(1) To control gradient in either natural or constructed channels.

(2) To serve as inlet or outlet structures for tile drainage systems in conjunction with gradient control.

(3) To control tailwater at the outlet of a spillway or conduit.

(4) To serve as reservoir spillways where the total drop (F) is relatively low.

Experience and comparison of structural and hydraulic characteristics show that drop spillways have certain advantages and disadvantages compared with other structures adapted to similar functional uses. These general advantages and disadvantages should not be regarded as a basis for final selection of the type of structure for a given site, but can be used in deciding whether the drop spillway should be one of the alternate types to be considered.

Advantages

(a) Stability. The drop spillway is very stable, and the likelihood of serious structural damage is more remote than for other types of structures.

(b) Nonclogging of weir. The rectangular weir is less susceptible to clogging by debris than the openings of other structures of comparable discharge capacity.

(c) Low maintenance costs. Drop spillways indicate a definite tendency toward lower maintenance cost as compared with other types of structures for most embankment and foundation soils.

(d) Ease and economy of construction. Drop spillways are relatively easy to construct. When reinforced concrete is used, the flat slabs and straight, plane-surfaced walls simplify the forming and steel setting operations. Standard form panels or reusable sectional forms may be used.

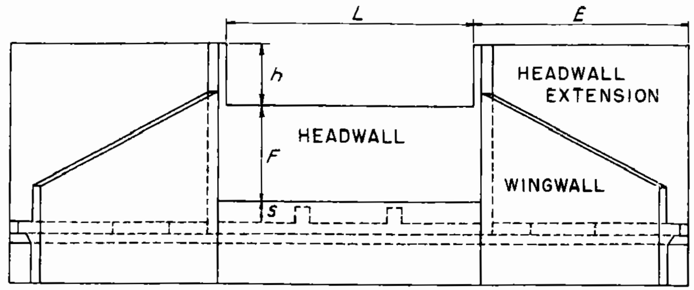
(e) Standardization. Drop spillways may be standardized readily both as to structural design and construction, which results in savings in engineering and construction costs.

Disadvantages

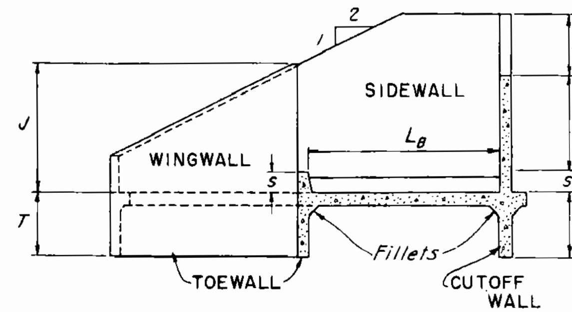
(a) The drop spillway may be more costly than some other types of structures where the required discharge capacity is less than 100 cfs and the total head or drop is greater than 8 or 10 feet.

(b) The drop spillway is not a favorable structure where it is desired to use temporary spillway storage to obtain a large reduction in discharge at and downstream from the structure. Discharge through a weir increases with the 1.5 power of the specific head at the weir (H_e) while the discharge through a closed conduit flowing full increases as the 0.5 power of the total drop in hydraulic grade line. The above statements are not to be taken as meaning that significant spillway storage should be neglected in determining the required discharge capacity of a drop spillway.

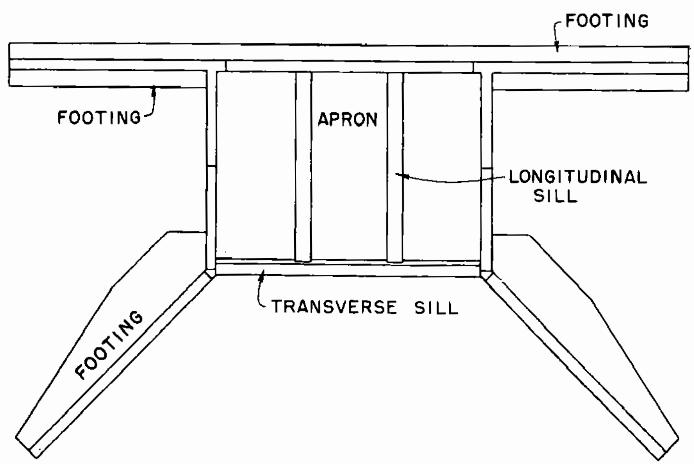
DROP SPILLWAYS: NOMENCLATURE AND SYMBOLS OF DROP SPILLWAY



DOWNSTREAM ELEVATION



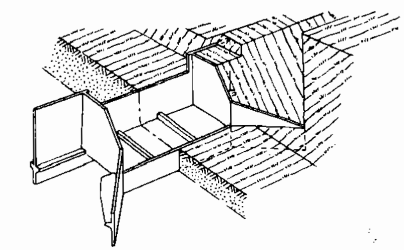
SECTION ON CENTER LINE



PLAN

SYMBOLS

- L = Length of weir.
- h = Depth of weir.
- F = Drop through spillway from crest of weir to top of transverse sill.
- s = Height of transverse sill.
- L_B = Length of apron.
- T = Depth of toewall below top of apron.
- C = Depth of cutoff wall below top of apron.
- d_c = Critical depth of weir.
- E = Length of headwall extension.
- J = Height of wingwall and sidewall at junction.



PERSPECTIVE VIEW

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-63 SHEET 1 OF 1 DATE 1-26-52
Rev. 12-14-53		

2. LAYOUT

General. The site selection and proportioning of a structure should be such that it satisfies the objectives and meets the stability requirements at minimum cost.

Site Selection. Proper site selection is dependent upon the availability of adequate field surveys and foundation data on all practicable alternate sites. The extent of field surveys required to prepare the most logical layout will depend upon the complexity of conditions peculiar to the problem. In some cases particular attention must be given to the effect of the proposed work on adjacent highways and their drainage structures, railroads, pipe lines, and other improvements or property that might be affected.

Comparative cost estimates probably will be necessary to determine the best layout. Volumes of earth fill and excavation, the cost of providing adequate protection during the construction period, volumes of concrete as affected by foundation conditions and other factors that vary from site to site, elevation of ground water, and other factors will affect costs. In the final selection, differences in cost at the various sites should be weighed against other advantages and disadvantages.

Channel Alignment. For gradient-control drops with definite approach channels, the site should be selected so that the spillway is located on a reasonably straight section of channel (on tangent), with neither upstream nor downstream curves within 100 to 200 feet of the structure. It often will be desirable to obtain straight alignment above and below the spillway by channel changes that merge smoothly with the existing channel. Modern earth moving equipment has made such channel changes practicable for many locations where, otherwise, poor alignment would have been unavoidable.

Poor upstream alignment, or any other disturbance that produces uneven distribution of velocity and discharge over the weir, is very apt to result in one or more of the following bad effects:

(1) Reduction in discharge capacity of the weir.

(2) Excessive scour of the earth embankment and channel banks just above the spillway.

(3) Uneven distribution of flow across the transverse sill at the end of the apron, and a reduction in energy dissipation by the apron and stilling pool of the structure.

(4) Excessive scour in the downstream channel just below the spillway apron and wingwalls, and downstream therefrom for a comparatively short distance.

The severity of these effects depends upon the extent of the upstream disturbance. Where the velocity of approach to the weir will be less than 2 feet per second throughout the anticipated life of the spillway, the effect of poor approach-channel alignment may be ignored. Where the approach velocity is apt to be higher, the approach channel must be straight.

Poor downstream alignment is not so serious as poor upstream alignment; however, it also should be avoided. Excessive scour is apt to develop if appreciable channel curvature exists immediately below the spillway. Such scour may be more severe than it would have been otherwise, due to the lack of complete dissipation of the overfall energy by the spillway apron and stilling pool.

The extent of the scour generated as the result of poor alignment will differ considerably from site to site for numerous reasons. It will be affected by the location, amount and rate of curvature, the velocity, depth of flow, duration of discharge, resistance of the channel bottom and banks to erosion, and perhaps other factors. Consequently, it will be difficult to predict the required extent of preventive riprap in advance of the scour. Where such predictions can be made, the riprap should be included in the original construction plans. It will be necessary to inspect such spillways after every significant storm and provide the riprap or other work necessary to protect and preserve them.

If, at a particular site, it is impracticable to avoid curvature, good upstream alignment must take precedence over desirable downstream conditions. In other words, drop spillways should be located so that the center line of a straight approach channel is coincident with the center line of the spillway.

Foundation Conditions. The site selected must provide an adequate foundation for the spillway. The foundation material must have the required supporting strength, resistance to sliding and piping, and be reasonably homogeneous so as to prevent differential or uneven settlement of the structure.

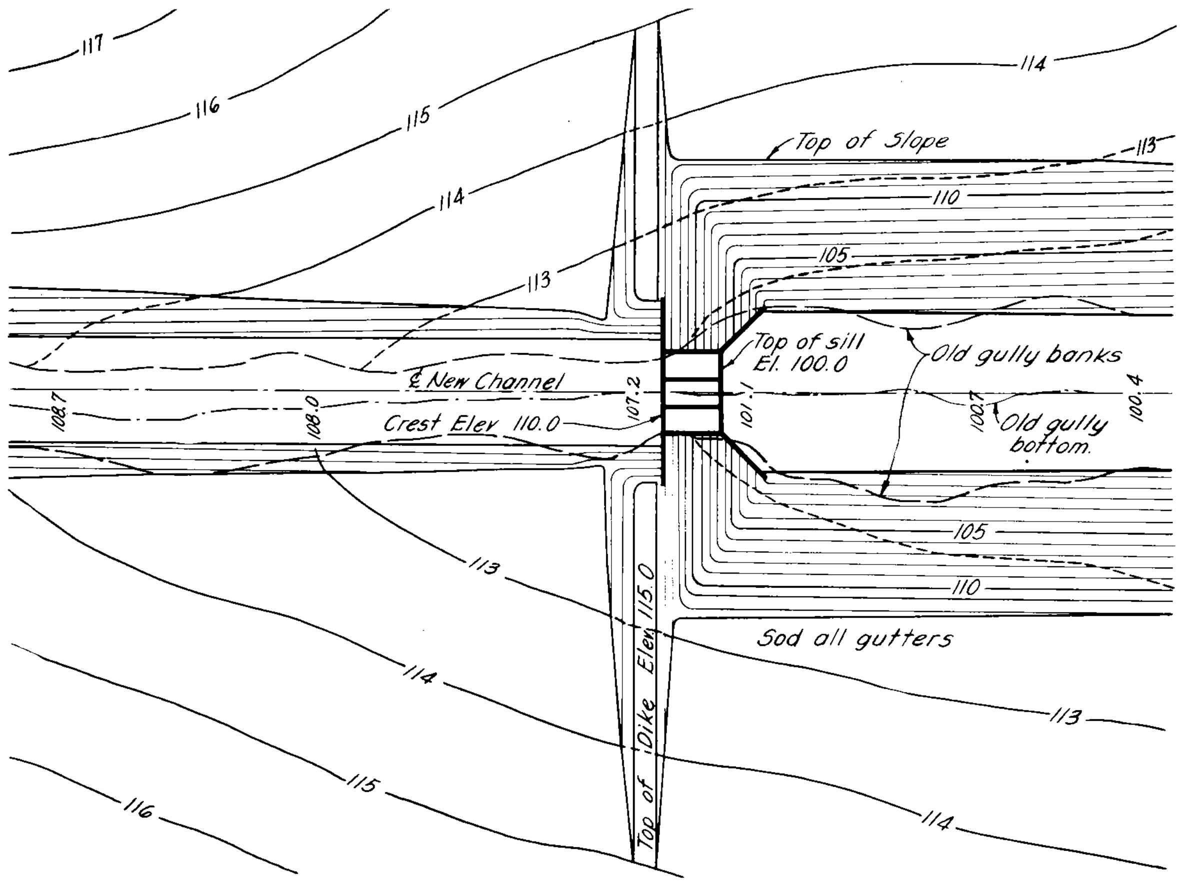
Piping, sliding, and vertical foundation loads are discussed elsewhere. Unequal settlement under various parts of the spillway must be carefully avoided; each part of the foundation must carry its proper share of the load. If the foundation materials vary appreciably as to consolidation under load over the foundation area, the reactions will not be distributed uniformly and cracking and differential settlement are probable.

Articulation of the various component parts of the structure may be desirable if foundation studies indicate only minor differences in foundation profiles. It usually will be wise to search for an alternate site or remove the foundation material and replace it with very carefully compacted homogeneous fill if the foundation profiles indicate appreciable differences that might lead to unequal settlement.

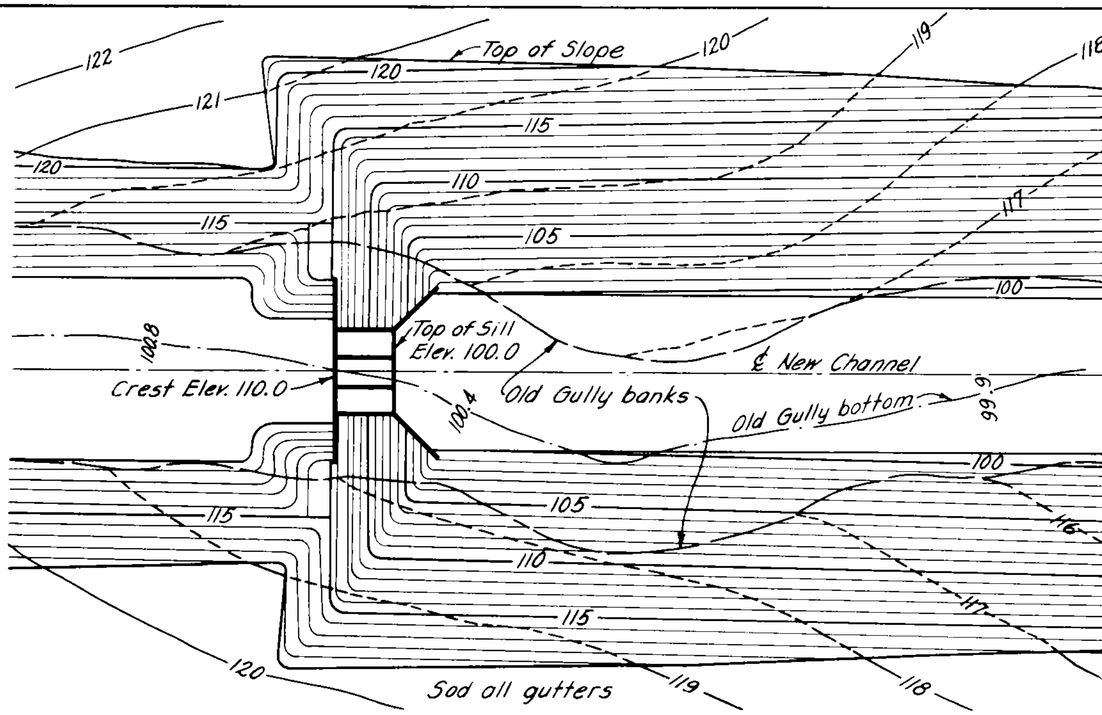
Obviously, foundation investigations must be made at each site. The extent of the investigation should depend upon the size and importance of the structure, known facts concerning the geology of the area, and the findings in the first borings or test pits. The investigations may range from visual classification of soils in one or two test holes, for structures 10 feet or less in height and of low failure hazard, to extensive soil borings, test pits, and soil-mechanics laboratory studies on higher structures where the hazard to life or property is significant should they fail.

Other Considerations. Other factors requiring investigation during the selection of the structure site are: (a) conditions that will affect the type and degree of protection against damage from runoff during

DROP SPILLWAYS: TYPICAL LAYOUTS



TOP OF HEADWALL EXTENSION ABOVE GULLY BANKS



TOP OF HEADWALL EXTENSION BELOW GULLY BANKS

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE Robert M. Salter, Chief ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES – 70 SHEET 1 OF 1 DATE 4-5-52

construction (b) material available for earth fill and the required volume of such fill (c) farming operations on the land adjacent to the site (d) roads, railroads, pipelines, and other structures that may affect or be affected by the structure.

Structure Dimensions. The determination of the dimensions of the various parts of the drop spillway is discussed in the subsections dealing with the hydraulic and structural design.

Top Width of Earth Embankment. The top width of the earth embankments should be such that they can be constructed and finished with the standard earth moving equipment. In the majority of cases, this sets the minimum top width at 8 feet. For gradient-control drop spillways without permanent pools of water above them, this minimum top width is sufficient for stability. Where a drop spillway acts as a reservoir spillway, the dikes are usually high for a considerable portion of their length and are in contact with the permanent pool. In such cases, the minimum top width of the dikes is given by the following equation which is applicable for heights of embankment up to 50 feet:

$$W = \frac{H + 35}{5} \qquad 2.1$$

where W = minimum top width in feet

H = maximum height of embankment in feet for range in values of 5 to 50 feet

Example: Given a maximum height of fill of 22 feet, find the minimum top width. $W = (22 + 35) \div 5 = 11.4$; hence, use a top width of 12 feet unless some other design consideration requires a larger value.

For practical construction reasons, no attempt should be made to vary the top width of the embankment as the fill height varies; select one top width and use it for the full length of the embankment.

Fill Slopes. The recommended fill slopes are: (a) for fill adjacent to the structure, not steeper than 2 horizontal to 1 vertical; (b) for earth embankment, 3 horizontal to 1 vertical where practical, with a minimum of 2 to 1. These 3-to-1 slopes are recommended not only for stability, but because they will facilitate maintenance operations.

Required Height of Earth Fill Above the Top of the Headwall Extension. For gradient-control drop spillways, the top of the settled earth fill should be at least 1 foot above the top of the headwall extension. Where a reservoir exists above the spillway, the top of the settled earth fill should be higher than the top of the headwall extension by an amount equal to the average depth of frost penetration, taken from fig. 2.1 (page 2.5), but not less than 1 foot. In the western part of United States, local experience on the average of maximum annual frost depths will need to be gathered to supplement the data in fig. 2.1 (page 2.5).

Riprap of Approach Channel. Field experience and observation of laboratory tests indicate that earth backfill, just above the crest of a weir and at the end of embankments adjacent to the ends of the weir opening, will be scoured out and carried away by discharges that approach design values unless it is protected by adequate riprap or vegetation. The depth and duration of discharge, erodability of backfill, alignment of approach channel, contraction at ends of the weir, sediment being transported, and density, vigor and type of vegetation on the backfill, and probably other factors affect the need for riprap.

Several drop spillways have been observed at which the channel grade line above the weir has been raised by an accumulation of sediment in dense, vigorous vegetation. Such a buildup of the channel results in reduced capacity of the weir.

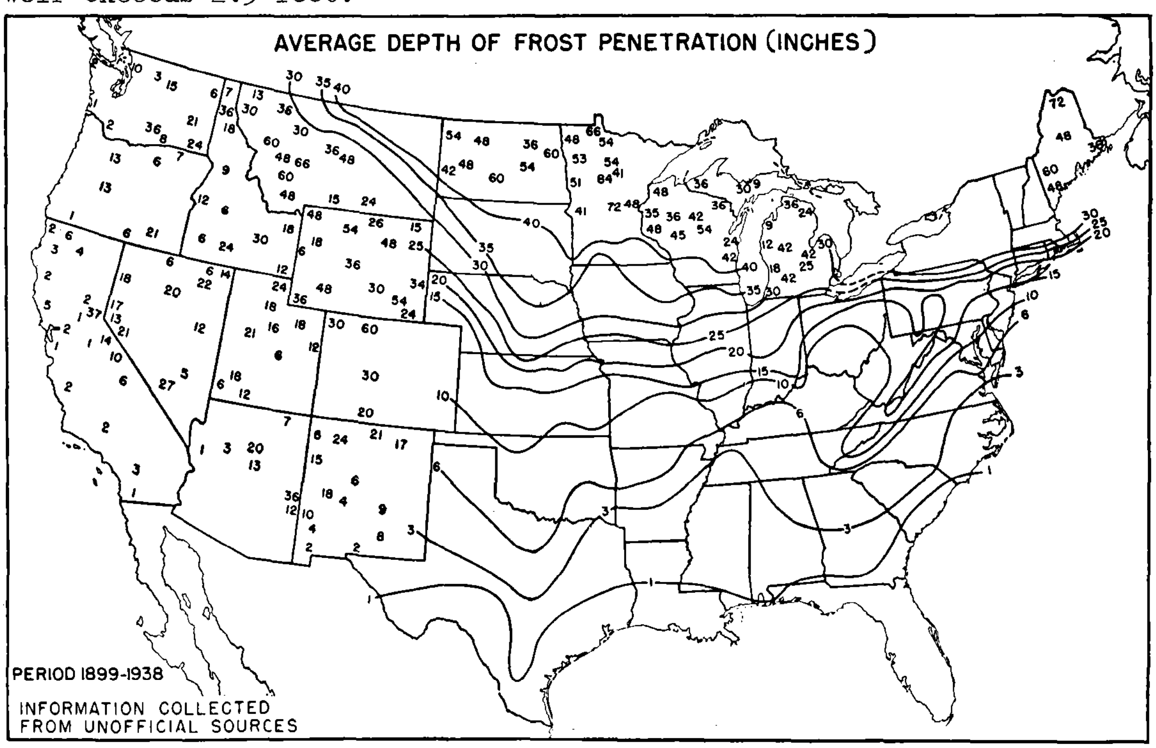
On other spillways, serious deep scour developed just above the weir, and especially at the ends of the weir, where additional turbulence is created by contraction of the flow. The fill just above a weir may be stable for several years, scour under a high discharge, and subsequently build up to original grade on the receding stage or by sedimentation during a series of low discharges.

It is highly desirable to avoid both scour and buildup above the headwall. Properly designed and placed riprap provides very good protection against both of these hazards.

Drop spillways that are located immediately below retarding dams will operate at or near design capacity more often and for longer time intervals than average gradient-control drops. Hence, the hazard from scour will be serious and the need for riprap is apparent. Drop spillways used as gradient-control structures in irrigation canals suffer relatively severe flow conditions and should always be riprapped above the weir.

Recommendations on the layout and requirements of riprap for drop spillways are presented in drawing ES-79 (page 2.6).

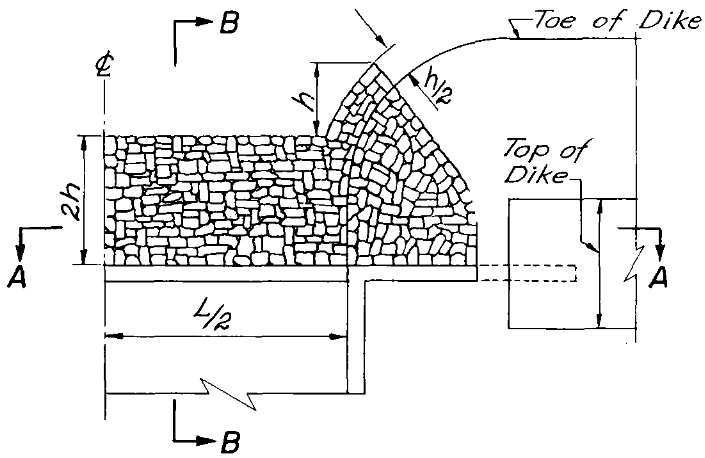
Riprap, placed in accordance with drawing ES-79 (page 2.6), should be considered as "must" protection in all cases where the depth of the weir exceeds 2.5 feet.



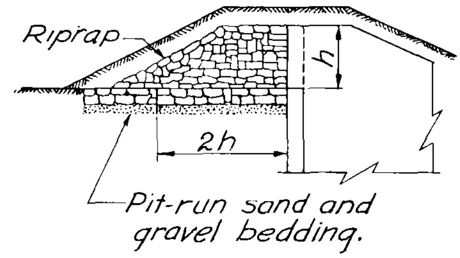
From "Climate and Man", Yearbook of Agriculture - 1941, p. 747

FIGURE 2.1

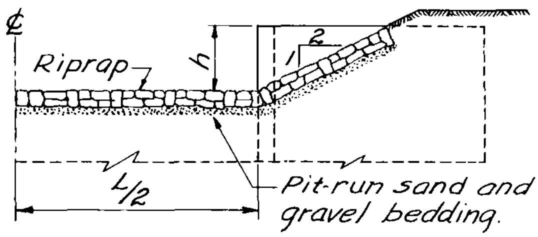
DROP SPILLWAYS– RIPRAP OF APPROACH CHANNEL– LAYOUT AND REQUIREMENTS



HALF PLAN



SECTION B–B



SECTION A–A

NOTES:

The riprap material should be of hard, durable stone or broken concrete with a unit weight equal to or greater than 150 pounds per cubic foot.

Angular, fragmented rock is preferable to rounded stone.

At least 75% of the riprap, by weight, should consist of pieces of rock or concrete, which equal or exceed the weight given in the table opposite the required depth of weir.

The thickness of the layer of riprap should be at least equal to the average diameter of rock, D, indicated in the table.

The riprap should be placed on a bed of coarse pit-run sand and gravel. The minimum thickness of the bedding is indicated in the table.

The spaces between the large rock of the riprap should be filled with spalls, smaller rock, and pit-run material.

The dimensions of the area of riprap shown in the above sketches are minimum dimensions.

The surface of the riprap should be as smooth as possible.

Depth of weir, h., in feet.	Average diameter. of Rock, D., in inches..	Weight of rock in pounds.	Minimum thickness of bedding in inches.
1.0	3.0	2.0	3.0
1.5	3.0	2.0	3.0
2.0	4.0	3.0	3.0
2.5	5.0	6.0	3.0
3.0	6.0	10.0	3.0
3.5	7.0	16.0	3.5
4.0	8.0	23.0	4.0
4.5	9.0	32.0	4.5
5.0	10.0	44.0	5.0
5.5	11.0	59.0	5.0
6.0	12.0	76.0	6.0

The average diameter, D., is defined as the diameter of a spherical rock of equal weight and density.

3. HYDRAULIC DESIGN

Hydrologic Determinations. Methods of determining peak rates and inflow hydrographs of runoff are discussed in Hydrology, Section 4 of the Engineering Handbook.

Selection of the frequency of the design flood flow for a particular drop spillway should be based on an evaluation of the following factors:

(1) Intended life of the structure

(2) Probable extent of damage, should the spillway fail due to lack of discharge capacity.

(3) Relative size and cost of the structure.

The discharge characteristics of a weir are such that a relatively large percentage increase in discharge capacity can be provided for a small percentage increase in total cost of the structure. The spillway cost is only a part of the total cost of the structure.

Discharge Capacity Determinations. Two general cases are encountered. They are:

(1) Those cases where the required discharge capacity and the total drop through the spillway, F, are known, and the problem is to choose the length and depth of weir to provide the required capacity, maintain an adequate freeboard, and provide economical proportions for the spillway.

(2) Those cases in which the dimensions of the structure are known, and it is necessary to know the discharge of the spillway operating with adequate freeboard or at maximum capacity.

In either case, the flow may be either free or submerged. Both free and submerged flow are discussed later.

Free Discharge. The discharge capacity of an aerated, rectangular weir without submergence is given by the formula

$$Q = CL\left(H + \frac{v_a^2}{2g}\right)^{3/2} \qquad 3.1$$

where Q = discharge in cfs

L = length of weir in ft

H = head on weir in ft (see fig. 3.1, page 3.4)

v_a = mean velocity of approach in fps

g = acceleration of gravity in ft per sec^2

C = discharge coefficient

A completely aerated weir is one in which unlimited quantities of air have free access to the space between the nappe and the headwall. Under such conditions, the nappe will be subject to atmospheric pressure on both upper and under surfaces. Ordinarily, complete aeration will not be attained and some small permissible differential in pressure, below atmospheric, will exist under the nappe. Provision must be made in the design for admission of air to the underside of the nappe by the forced development of end contractions or by air vents. Failure to provide aeration will lead to the formation of excessive negative pressures (below atmospheric) under the nappe which in turn cause fluctuation of head, instability of flow, and increased load on the headwall.

Drawing ES-81 (page 3.3) gives the design equations and procedure for estimating the size of air vents required for drop spillways. If the differential pressure, p, exceeds 0.3 ft. of water, the increased loads on the headwall should be included in the structural analysis and design.

Only a limited investigation of the discharge capacity of drop spillways has been made. The lack of test data, and consideration of the varied conditions under which these structures will operate, lead to a recommended design value for $C = 3.1$. Use of this coefficient is based on the assumption that flow approaching the weir is at subcritical velocities, i.e., the depth of flow is greater than critical depth.

Contraction at the ends of the weir has not been treated specifically because of a lack of data applicable to the structures under discussion and because of its small effect on drop spillways of usual proportion.

It is believed that the use of the coefficient $C = 3.1$ is sufficiently conservative to have made reasonable allowance for possible end contractions and other indeterminate factors that affect the discharge capacity.

Velocity of Approach. The total energy head producing flow over the weir is equal to $H + (v_a^2 \div 2g)$. The section in the approach channel at which H and the approach velocity, v_a, are estimated should meet the following conditions: (Study fig. 3.1, page 3.4)

(1) It should be 3H or more upstream from the weir, so as to be above any significant influence of the drop-down curve which results from the increase in velocity as the flow approaches the weir.

(2) For simplicity in computations, it should be upstream from any constrictions of the approach channel that may be imposed by an earth embankment which diverts the flow to the weir.

(3) It should not be so far upstream that the energy losses between the chosen section and the weir will affect the design significantly. In other words, the assumption of a level energy line from the chosen section to the weir must be reasonably correct.

The section in question may be chosen at any location which meets conditions 1 and 3, listed above. The actual cross section determined by field measurement may be regular or irregular in shape; if irregular, an equivalent trapezoidal section may often be selected to facilitate computations.

The velocity of approach is equal to the discharge divided by the cross-sectional area of the chosen section. $v_a = Q \div a_a$

DROP SPILLWAYS — AERATION OF WEIRS

EQUATION

$$\frac{A}{L} = 5.3 \times 10^{-4}\ \frac{H_e^{3.64}}{p^{1.64}}$$

where:

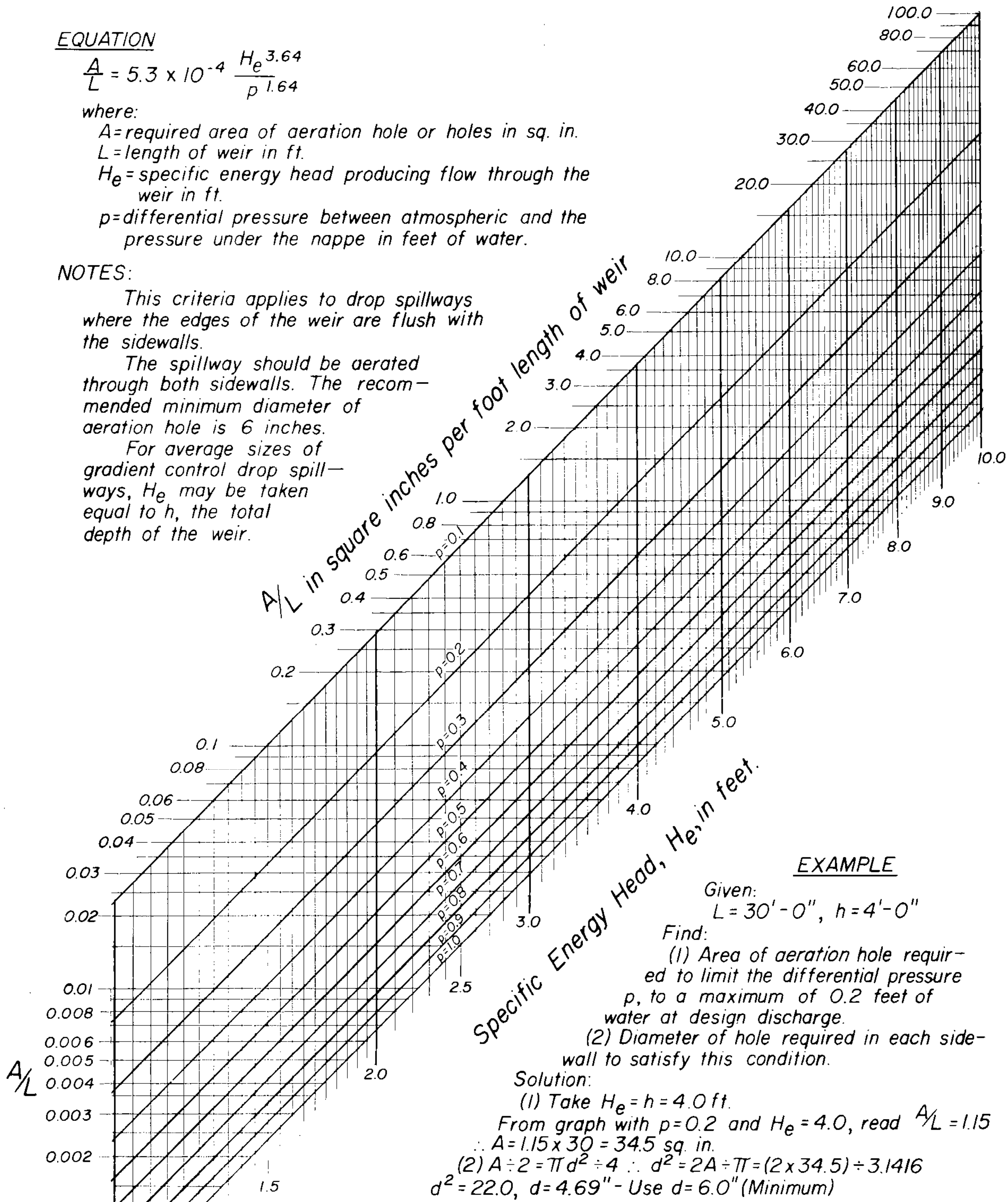
A = required area of aeration hole or holes in sq. in.
L = length of weir in ft.
H_e = specific energy head producing flow through the weir in ft.
p = differential pressure between atmospheric and the pressure under the nappe in feet of water.

NOTES:

This criteria applies to drop spillways where the edges of the weir are flush with the sidewalls.

The spillway should be aerated through both sidewalls. The recommended minimum diameter of aeration hole is 6 inches.

For average sizes of gradient control drop spillways, H_e may be taken equal to h, the total depth of the weir.

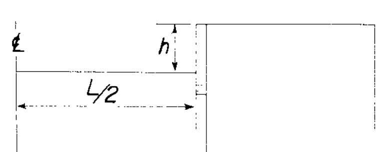


EXAMPLE

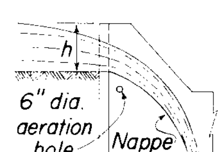
Given:
$L = 30'\text{-}0''$, $h = 4'\text{-}0''$

Find:
(1) Area of aeration hole required to limit the differential pressure p, to a maximum of 0.2 feet of water at design discharge.
(2) Diameter of hole required in each sidewall to satisfy this condition.

Solution:
(1) Take $H_e = h = 4.0$ ft.
From graph with $p = 0.2$ and $H_e = 4.0$, read $A/L = 1.15$
$\therefore A = 1.15 \times 30 = 34.5$ sq. in.
(2) $A \div 2 = \pi d^2 \div 4 \ \therefore\ d^2 = 2A \div \pi = (2 \times 34.5) \div 3.1416$
$d^2 = 22.0$, $d = 4.69''$ - Use $d = 6.0''$ (Minimum)



HALF FRONT ELEVATION

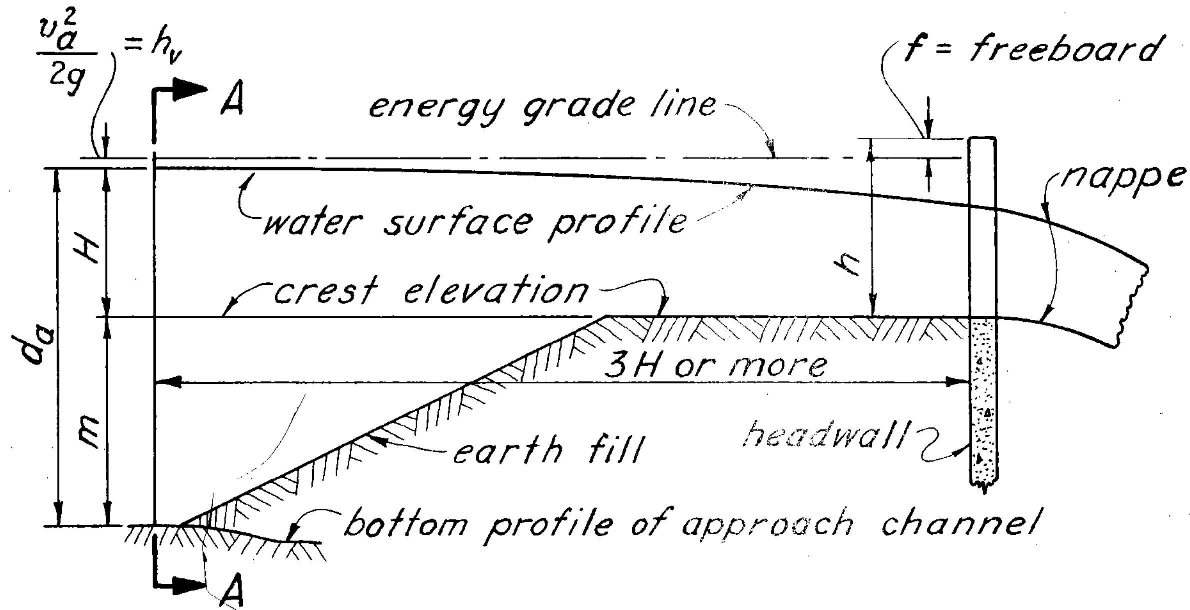


SIDE ELEVATION

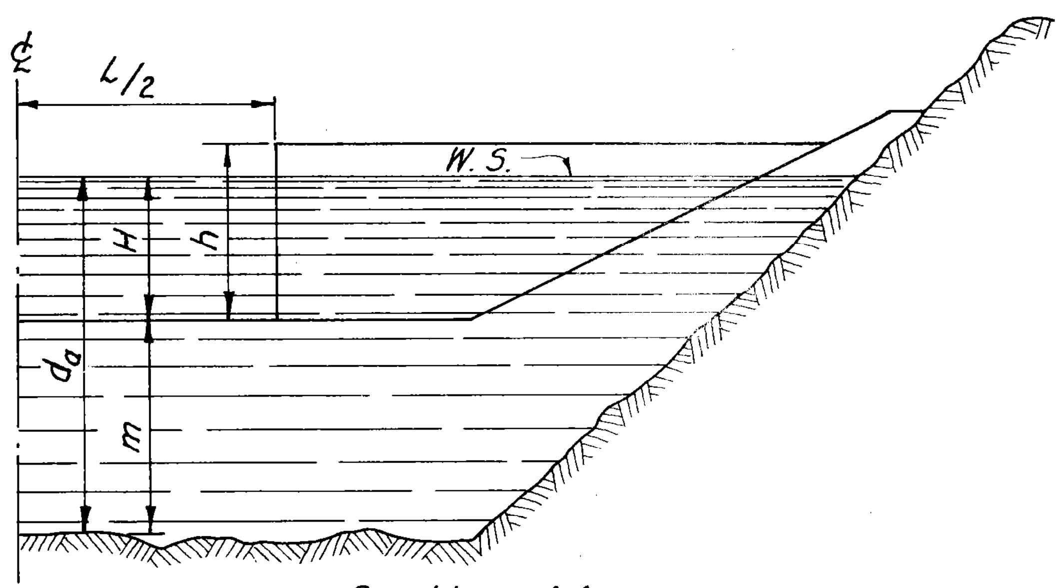
REFERENCE:
Aeration of Spillways by G. H. Hickox. — Trans. ASCE 1944, page 537, paper No. 2215

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING DIVISION - DESIGN SECTION

STANDARD DWG. NO.
ES-81
SHEET 1 OF 1 SHEETS
DATE 7-10-53



Condensed Profile on ℄ Spillway



Section AA

Half Cross Section of Approach Channel showing Projected Weir Opening

FIGURE 3.1

From equation 3.1 (page 3.1) the discharge is a function of $H + (v_a^2 \div 2g)$, so that in the determination of weir dimensions it is necessary only to determine the sum of H and $(v_a^2 \div 2g)$ since their sum is all that is required to determine discharge.

If, for some reason, it is necessary to know the upstream stage-discharge curve for such a weir, it can be found by the following procedure.

Step 1. Assume various discharges and compute the sum $H + (v_a^2 \div 2g) = [Q \div (CL)]^{2/3}$ from equation 3.1 (page 3.1).

Step 2. Determine m for section AA from physical measurement for the problem at hand and add it to the values of $H + (v_a^2 \div 2g)$ obtained in step 1 to get the specific energy at the section. $H_e = d_a + (v_a^2 \div 2g) = m + H + (v_a^2 \div 2g)$ (see fig. 3.1, page 3.4); m will be positive if channel bottom is below crest elevation at section AA and negative if above.

Step 3. For each value of H_e determined in step 2, find the velocity head, $(v_a^2 \div 2g)$, d_a and H at section AA by systematic trial and error. This step is explained best by an example as follows:

Example 3.1

Known:

1. Channel dimensions at section AA: bottom width, $b = 40.0$ ft; side slopes 2 to 1, or $z = 2$; $m = -0.10$ ft (i.e., bottom of approach channel is above crest)

2. Weir dimensions: $L = 30.0$ ft; $h = 5.0$ ft

3. Discharge, $Q = 905$ cfs

4. Coefficient of discharge, $C = 3.1$

Find:

1. Velocity head at section AA

2. Velocity of approach, v_a, at section AA

3. H = stage of water surface above crest of weir at section AA.

Procedure:

Substep 1. $H + (v_a^2 \div 2g) = [Q \div (CL)]^{2/3} = [905 \div (3.1 \cdot 30)]^{2/3} = 4.56$ ft

Substep 2. $H_e = m + H + (v_a^2 \div 2g) = -0.10 + 4.56 = 4.46$ ft

Substep 3. Prepare a table as follows: Assume trial values of d_a and for each such assumed value, compute a_a, v_a, $(v_a^2 \div 2g)$, and H_e and compare with actual value of H_e obtained in step 2. Interpolate to get the value of d_a which is associated with the actual value of H_e and then compute the velocity head $(v_a^2 \div 2g) = H_e - d_a$.

Trial Value of d_a	a_a	$v_a = \frac{Q}{a_a}$	$\frac{v_a^2}{2g}$	$H_e = d_a + \frac{v_a^2}{2g}$	Remarks
3.80	180.9	5.00	0.39	4.19	too low
4.00	192.0	4.71	0.35	4.35	too low
4.20	203.3	4.45	0.31	4.51	too high
4.14[1]	199.9	4.53	0.32	4.46	check

Interpolate

$$d_a = 4.00 + \left(\frac{4.46 - 4.35}{4.51 - 4.35}\right) \cdot (4.20 - 4.00) = 4.14$$

This computation can be tabulated easily as follows:

$$\begin{array}{lll} 4.51 \text{ --- } 4.20 & & \\ 4.46 & & \\ \underline{4.35} \text{ --- } \underline{4.00} & & 4.00 \\ \frac{11}{16} \circ 0.20 = & & \underline{0.14} \\ & & 4.14 \end{array}$$

[1]Check made after interpolation. Thus $(v_a^2 \div 2g) = H_e - d_a =$ $4.46 - 4.14 = 0.32$ ft and $v_a = 4.53$ fps.

Substep 4. With m and d_a known, compute H from the equation $H = d_a - m$. In this example $H = 4.14 - (-0.10) = 4.24$ ft.

Step 4. Plot values of H against Q to give the required stage-discharge curve.

Solution of several examples will demonstrate that where a reservoir full of water exists above the spillway without an excavated approach channel to the weir, the velocity of approach may be ignored. Then H can be computed directly from equation 3.1 (page 3.1) with $(v_a^2 \div 2g) = 0$, or $H = [Q \div (CL)]^{2/3}$ In all other cases the velocity of approach should be included in the analysis.

<u>Freeboard</u>. Freeboard is the vertical distance from the maximum water-surface elevation on the upstream side of the headwall extension to the top of the headwall extension for peak design discharge over the weir. It is a safety factor to provide against possible occurrence of conditions, not anticipated during the design, that would decrease the capacity of the spillway or increase the discharge requirements and to provide protection against overtopping by wave action where it can take place.

Most of the velocity head that exists at the section where H is measured (see fig. 3.1, page 3.4) will be converted to elevation head along the headwall extensions where the stream lines impinge against it. Since the energy grade line may be assumed level between section AA, fig. 3.1 (page 3.4), and the weir, and since most of the approach velocity head is regained at the headwall extension, the total weir depth is given by the equation

$$h = f + H + \frac{v_a^2}{2g} \qquad 3.2$$

where h = total depth of weir in ft

f = freeboard in ft

and other terms are as previously defined.

Where wave action will not occur, it is convenient and logical to consider freeboard in terms of increased weir discharge capacity. It also seems logical to assume that the required freeboard should be some function of the overfall through the drop spillway, F, since the possible damage due to failure increases with an increase in F.

Following this line of reasoning, let the maximum discharge capacity of the weir without freeboard be $Q(1 + \delta)$. Then from equations 3.1 (page 3.1) and 3.2 (page 3.7)

$$Q(1 + \delta) = CLh^{3/2} = CL\left(f + H + \frac{v_a^2}{2g}\right)^{3/2} \qquad 3.3$$

where

δ = increase in discharge capacity of weir, expressed as a decimal, due to an increase in head on the weir equal to f.

A study of various functional relationships between δ and F led to the selection of the following reasonable equation

$$\delta = 0.10 + 0.01\ F \qquad 3.4$$

Substitution of δ from equation 3.4 into equation 3.3 gives

$$Q = \frac{CLh^{3/2}}{1.10 + 0.01\ F} \qquad 3.5$$

or

$$h = \left[\frac{Q(1.10 + 0.01\ F)}{CL}\right]^{2/3} \qquad 3.6$$

or

$$L = \frac{Q(1.10 + 0.01\ F)}{Ch^{3/2}} \qquad 3.7$$

The use of these equations will be discussed and illustrated later.

Where wave action will occur, the freeboard must be governed by anticipated wave height. Wave freeboard, f_w, is the difference in elevation between the reservoir water-surface elevation at design discharge and the top of the headwall extension. Wave height is related to wind velocity and the length of water surface subject to wind action, called length of exposure or fetch.

Stephenson's equation for wave freeboard is

$$f_w = 0.0206\ D^{1/2} - 0.117\ D^{1/4} + 2.5 \qquad 3.8$$

where f_w = wave freeboard in ft

D = length of fetch in ft

This equation requires excessive freeboard for low dams of low failure hazard. Hence, it has been modified to reduce the freeboard requirements for drop spillways with controlled head, F, of less that 20 feet.

The recommended equations are:

(1) For values of D equal to or less than 6000 ft and F equal to or less than 20 ft, use

$$f_w = 0.000095\ D + \frac{F^{1/2}}{2} + 0.27 \qquad 3.9$$

(2) For values of D greater than 6000 ft and F equal to or less than 20 ft, use

$$f_w = 0.0206\ D^{1/2} - 0.117\ D^{1/4} + \frac{F^{1/2}}{2} + 0.27 \qquad 3.10$$

(3) For values of D equal to or less than 6000 ft and F greater than 20 ft, use

$$f_w = 0.000095\ D + 2.50 \qquad 3.11$$

(4) For values of D greater than 6000 ft and F greater than 20 ft, use Stephenson's equation, number 3.8.

In the solution of equations 3.8 and 3.10, it is helpful to recognize that $D^{1/4} = (D^{1/2})^{1/2}$, i.e., the fourth root of D is equal to the square root of the square root of D.

Equations 3.9 and 3.10 have been plotted in fig. 3.2 (page 3.9).

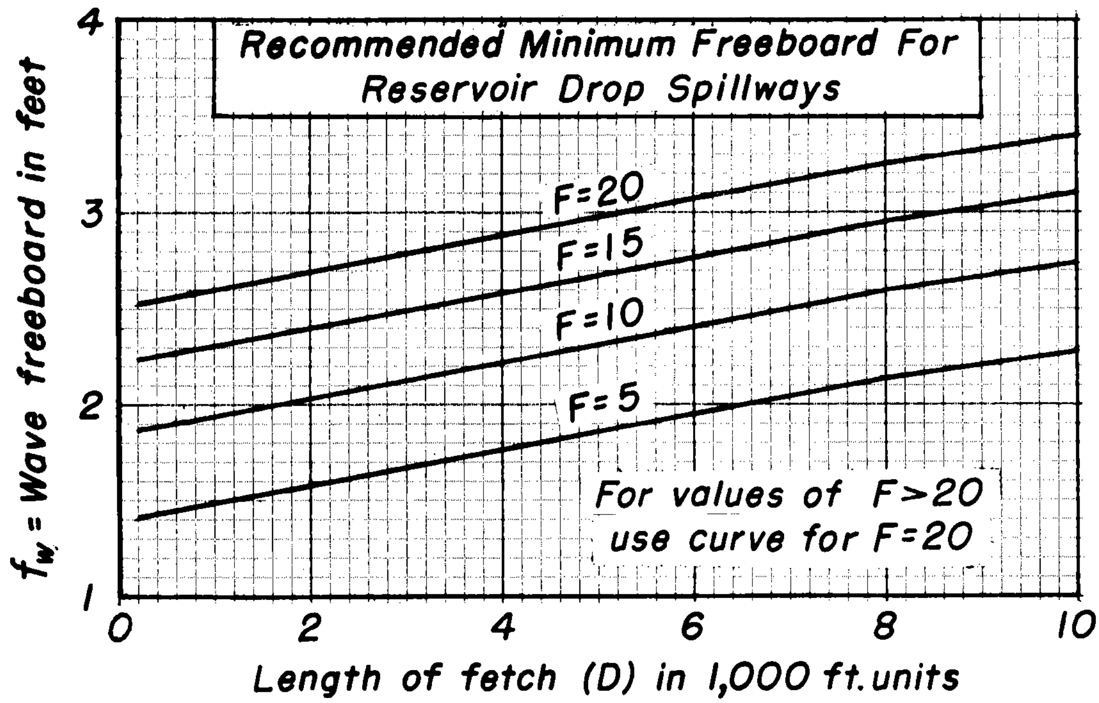


FIGURE 3.2

Example 3.2

Given: F = 12 ft; D = 3600 ft

Find: Required wave freeboard, f_w

Solution: Since D is less than 6000 ft, substitute given data in equation 3.9 (page 3.8) and solve for f_w as follows

$$f_w = (0.000095 \cdot 3600) + (12^{1/2} \div 2) + 0.27$$
$$= 0.34 + (3.46 \div 2) + 0.27 = \underline{2.34\ \text{ft}} \quad \text{Ans.}$$

Note that this answer can be read directly from fig. 3.2 with sufficient accuracy by eye interpolation.

<u>Working Procedures, Tools, and Examples for Free Flow</u>. The usual design problem of selecting a length and depth of weir to discharge a certain required peak rate of flow is greatly facilitated by the use of drawing ES-65 (page 3.11).

Drawing ES-65 (page 3.11) provides a solution of equation 3.5 (page 3.7) in which the freeboard is a function of the controlled head as defined by equations 3.3 (page 3.7) and 3.4 (page 3.7). It has been prepared to cover the most commonly encountered range in the variables F, h, L, and Q. Where the range of variables on drawing ES-65 (page 3.11) does not cover the situation at hand, equation 3.5 (page 3.7) or one of the equations 3.6 or 3.7 (page 3.7) must be used.

When it is desirable to use a greater freeboard than provided for by equation 3.5 (page 3.7), as for example in a reservoir drop spillway, the required freeboard is determined and added to the value of $H + (v_a^2 \div 2g)$ (see equation 3.2, page 3.7), which is determined from equation 3.1 (page 3.1), for the required discharge and an assumed trial value of L. To arrive at reasonable and economical weir proportions, it probably will be necessary to select several trial values of L and compute the required total weir depth, h, for each and then select the particular combination of L and h that will carry the required discharge with the desired freeboard and produce the lowest-cost structure adaptable to the site under study.

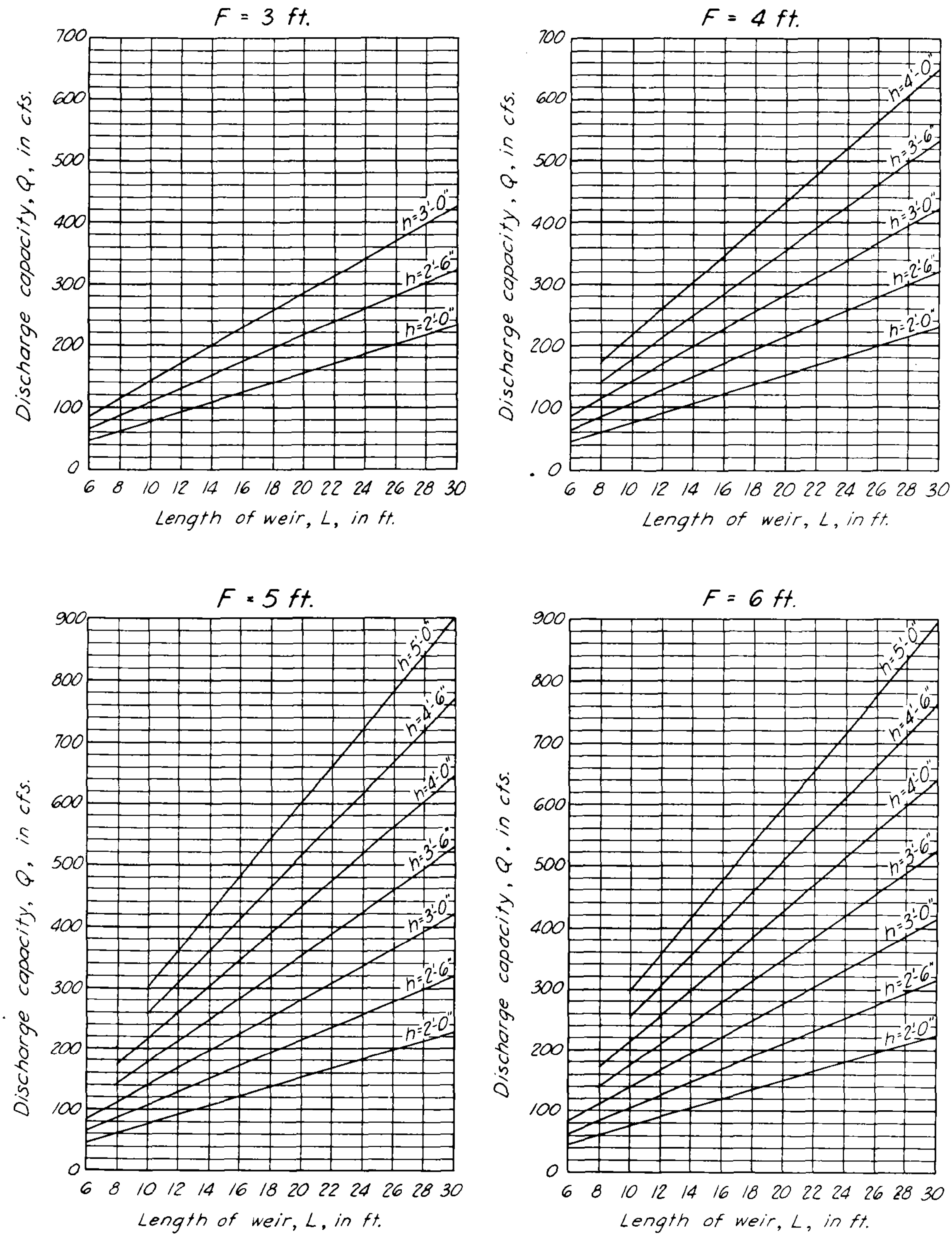
The spillway with the lowest volume of concrete is not necessarily the one which, when combined with the other items of cost, will produce the lowest cost for the entire structure, including excavation, foundation preparation, hand-compacted backfill, earth embankment, and other possible cost items. Carefully prepared cost estimates for the complete structure are necessary for the selection of the best weir proportions. Even after such comparisons have been made, other practical considerations may lead to final selection of a structure other than the one indicated by cost estimates as having the lowest installation cost. In any event, comparative cost estimates are essential as a guide to judgment.

<u>Example 3.3</u>

Given:
1. Required discharge capacity, Q = 340 cfs
2. Net drop, F = 8 ft
3. Free flow condition (unsubmerged)
4. Use minimum freeboard as defined by equations 3.3 and 3.4 (page 3.7)
5. Coefficient of discharge, C = 3.1

Find:
1. Reasonable combinations of length of weir, L, and depth of weir, h, that will carry required discharge capacity and provide minimum freeboard.
2. Freeboard for one combination of L and h (to illustrate how this is done).

DROP SPILLWAYS: SOLUTION OF EQUATION $Q = \dfrac{3.1\ Lh^{3/2}}{(1.10 + 0.01\ F)}$

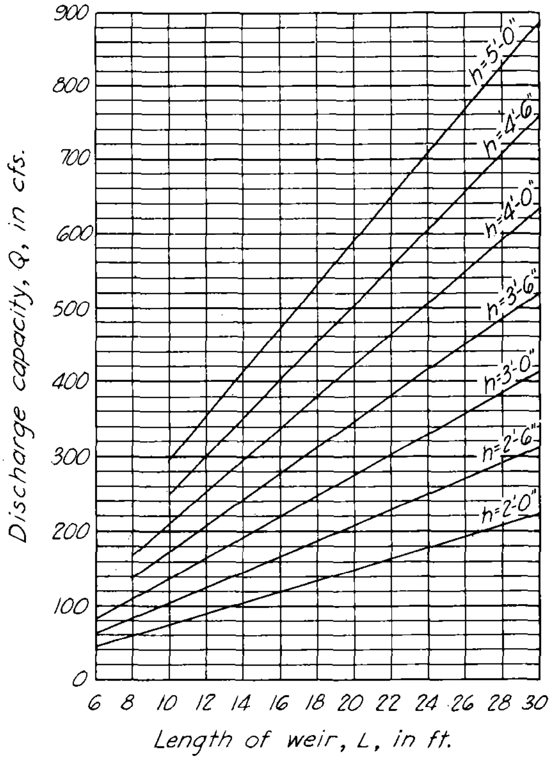


Note: h = total depth of weir, in feet (including treeboard)
F = net drop from crest to top of transverse sill, in feet
(For type B drops keep h ÷ F less than 0.75)

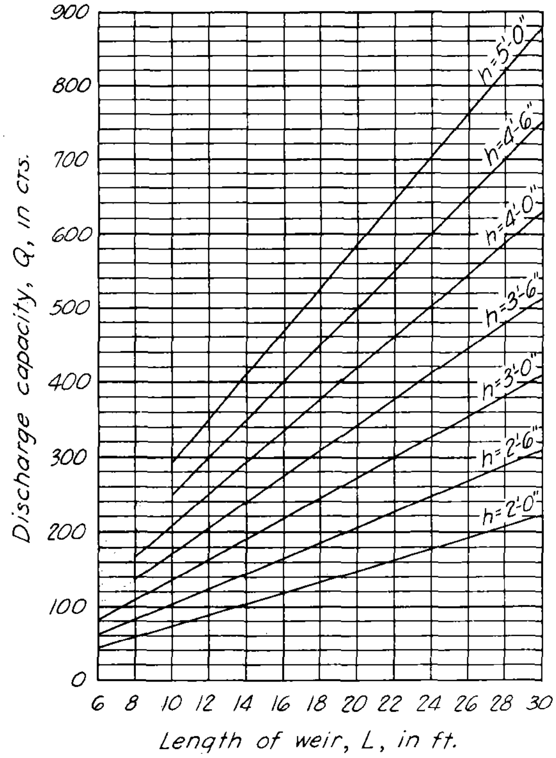
REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING STANDARDS UNIT	STANDARD DWG. NO. ES-65 SHEET 1 OF 2 DATE 2-8-52

DROP SPILLWAYS: SOLUTION OF EQUATION $Q = \dfrac{3.1\ L h^{3/2}}{(1.10 + 0.01\ F)}$

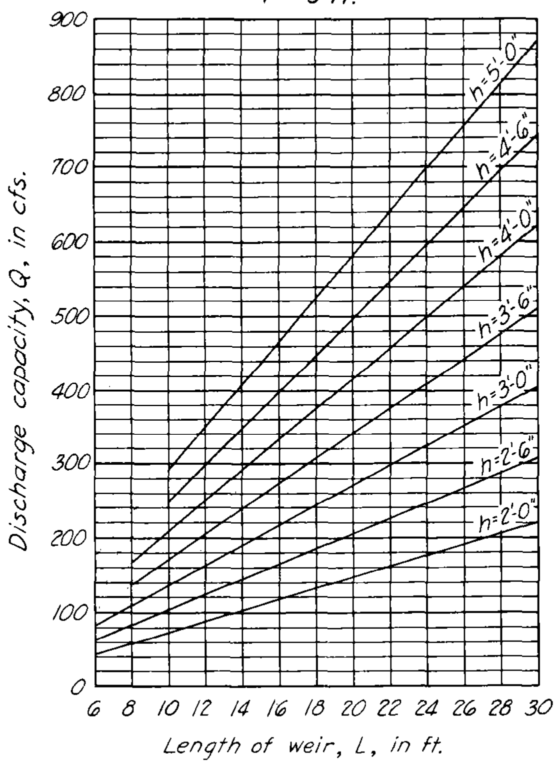
F = 7 ft.



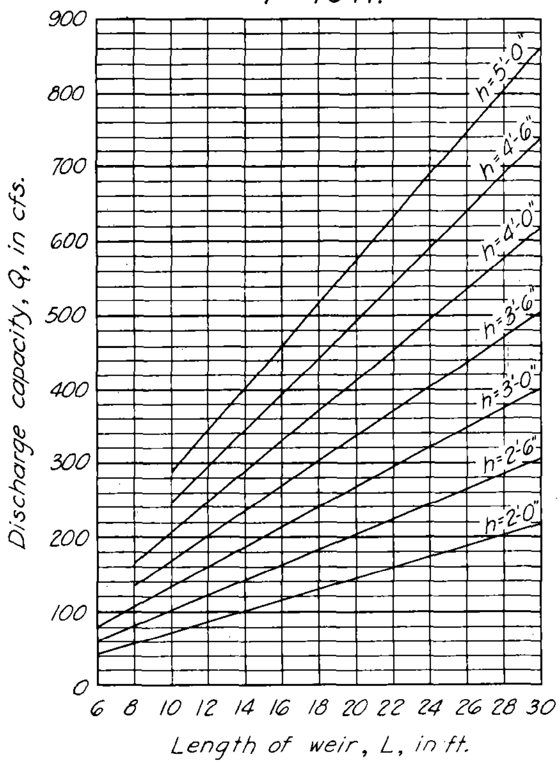
F = 8 ft.



F = 9 ft.



F = 10 ft.



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Solution: Use equation 3.7 (page 3.7) and substitute given values.

$$L = \frac{Q(1.10 + 0.01\ F)}{Ch^{3/2}} = \frac{340\ [1.10 + (0.01 \circ 8)]}{3.1\ h^{3/2}}$$

$$= 129.4 \div h^{3/2}$$

Next prepare a table as shown below; assume values of h and complete the computations indicated. Three-halves powers can be obtained from table 38, page 103 of King's "Handbook of Hydraulics," or from drawing ES-37, Engineering Handbook, Section 5 on Hydraulics.

1	2	3	4
h	$h^{3/2}$	$L = \frac{129.4}{h^{3/2}}$	Practical Values of L
3.0	5.20	24.9	25
3.5	6.55	19.8	20
4.0	8.00	16.2	16
4.5	9.55	13.6	14
5.0	11.18	11.6	12

To illustrate the method of finding the freeboard provided for a specific combination of L and h in the above table, choose $h = 3.00$ ft and the companion $L = 24.9$ ft. The freeboard is found by computing the head, $H + (v_a^2 \div 2g)$, necessary to carry the required discharge and subtracting this value from the depth of the weir, h.

From equation 3.1 (page 3.1)

$$H + \frac{v_a^2}{2g} = \left(\frac{Q}{CL}\right)^{2/3} = \left(\frac{340}{3.1 \cdot 24.9}\right)^{2/3} = 2.69$$

Then from equation 3.2 (page 3.7)

$$f = h - [H + (v_a^2 \div 2g)] = 3.00 - 2.69 = 0.31 \text{ ft}$$

For the practical value of $L = 25$ associated with $h = 3.00$, the value of f is found in the same way, $H + (v_a^2 \div 2g) = [Q \div (CL)]^{2/3} = [3.40 \div (3.1 \cdot 25)]^{2/3} = 2.68$ and $f = 3.00 - 2.68 = 0.32$ ft

Comment: It should be noted that columns 1 and 4 of the above table can be filled in for this case directly from drawing ES-65 (page 3.11). Of course, if either L or h is fixed by site or functional requirements, the other weir size variable may be found directly from equation 3.6 or 3.7 (page 3.7).

Example 3.4

Given:
1. Net drop, F = 15 ft
2. Free flow condition (unsubmerged)
3. Required discharge capacity, Q = 2460 cfs
4. Reservoir immediately above spillway with length of fetch = 1800 ft
5. Coefficient of discharge, C = 3.1

Find:
1. Wave freeboard required
2. Combinations of L and h t, will carry required discharge with the requ ed freeboard.

Solution: The required wave freeboard can be computed using equation 3.9 (page 3.8), or it can be read directly from fig. 3.2 (page 3.9). Substituting in equation 3.9 (page 3.8), we have

$$f_w = (0.000095 \cdot 1800) + (15^{1/2} \div 2) + 0.27 = 2.38 \text{ ft}$$

From equation 3.2 (page 3.7), $H + (v_a{}^2 \div 2g) = h - f$ and from equation 3.1 (page 3.1), $H + (v_a{}^2 \div 2g) = [Q \div (CL)]^{2/3}$; hence, for this case

$$h - f_w = \left(\frac{Q}{CL}\right)^{2/3}$$

or

$$L = \frac{Q}{C(h - f_w)^{1.5}} = \frac{2460}{3.1(h - 2.38)^{1.5}} = \frac{794}{(h - 2.38)^{1.5}}$$

With f_w, C, and Q known, assume values of h and compute L. Prepare a table as follows to facilitate the computations.

h	h − 2.38	$(h - 2.38)^{1.5}$	$L = \frac{794}{(h - 2.38)^{1.5}}$	Practical Value of L
7.00	4.62	9.93	80.0	80
7.50	5.12	11.59	68.5	69
8.00	5.62	13.32	59.6	60
8.50	6.12	15.14	52.4	53
9.00	6.62	17.03	46.6	47
9.50	7.12	19.00	41.8	42
10.00	7.62	21.03	37.8	38

Comment: For any selected companion set of weir dimensions, the stage-discharge curve can be determined by methods given in example 3.1 (page 3.5), the paragraph following example 3.1, and as explained in previous discussion.

Those cases where it is necessary to find the discharge capacity of a given weir operating with minimum acceptable freeboard can be solved by a direct application of equation 3.5 (page 3.7). This is illustrated by the following example.

Example 3.5

Given: 1. $h = 5.00$ ft; $L = 18.00$ ft

2. $F = 8$ ft

3. $C = 3.1$

4. Free flow conditions (unsubmerged)

Find: 1. Discharge capacity of the weir operating with minimum acceptable freeboard.

Solution: Substitute the given values in equation 3.5 (page 3.7) and compute Q.

$$Q = \frac{CLh^{3/2}}{1.10 + 0.01\ F} = \frac{3.1 \cdot 18 \cdot 5^{3/2}}{1.10 + (0.01 \cdot 8)} = 529 \text{ cfs}$$

Comment: For this case, the above resulting Q could have been read with sufficient accuracy directly from drawing ES-65 (page 3.11).

It should also be noted that the capacity of such a weir without freeboard = $Q(1 + \delta) = Q(1.10 + 0.01\ F)$. In this case $Q(1.10 + 0.01\ F) = 1.18\ Q = 1.18 \cdot 529 = 624$ cfs, or an 18 percent increase in discharge above that of the same weir operating with minimum freeboard.

The discharge capacity of a given weir operating with a fixed freeboard that is not dependent on F can be computed from equation 3.1 (page 3.1). This case is illustrated by the following example.

Example 3.6

Given: 1. $h = 5.00$ ft; $L = 18.00$ ft

2. $C = 3.1$

3. Free flow conditions (unsubmerged)

4. Wave freeboard, $f_w \doteq 1.80$ ft

Find: Discharge capacity of the weir operating with a freeboard of 1.8 ft

Solution: From equation 3.2 (page 3.7), $H + (v_a^2 \div 2g) = h - f$ $= 5.00 - 1.80 = 3.20$ ft, and from equation 3.1 (page 3.1), $Q = CL[H + (v_a^2 \div 2g)]^{1.5} = 3.1 \cdot 18 \cdot 3.20^{1.5} = 318$ cfs.

Submerged Discharge. No experimental data on submerged flow over drop spillways are available. The following material has been developed from a study of the reported test results of submerged flow over several types of weirs and over earth embankments. The data studied indicates a wide range in the effect of submergence. Hence, precise results should not be expected from submergence computations.

Submerged discharge is related to free discharge by the equations

$$Q_s = RQ_f \qquad 3.12$$

$$q_s = Rq_f \qquad 3.13$$

where Q_s = submerged discharge in cfs

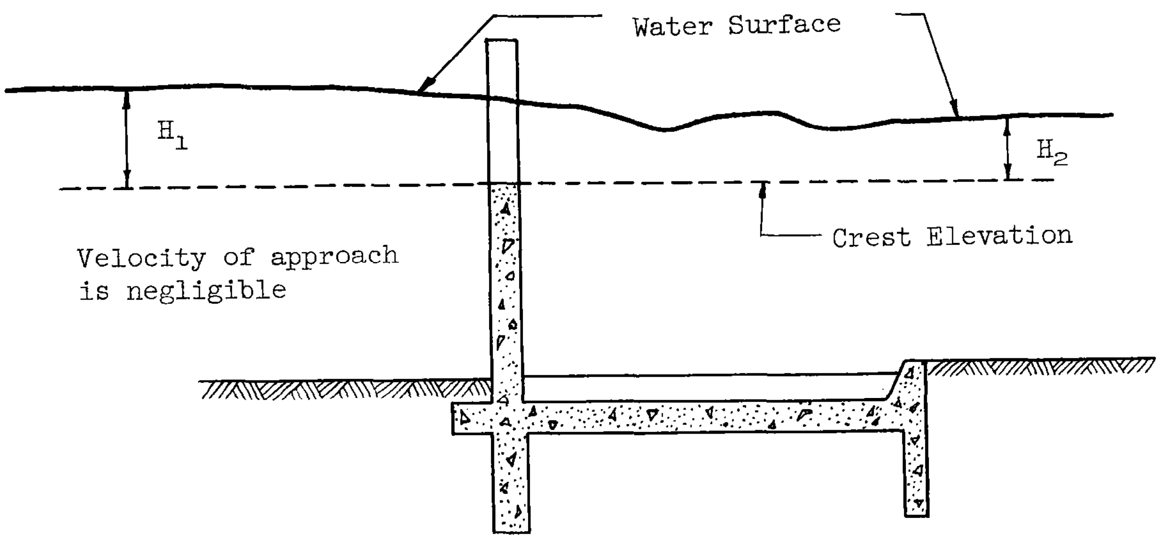
Q_f = free discharge in cfs

q_s = submerged discharge per foot length of weir in cfs = $Q_s \div L$

q_f = free discharge per foot length of weir in cfs = $Q_f \div L$

R = ratio as defined by equations 3.12 and 3.13 (page 3.15)

Analysis of available submergence data resulted in the preparation of fig. 3.4 (page 3.17) which gives the relationship between R and the ratio $(H_2 \div H_1)$ in graphical form. The values H_2 and H_1 are defined below and illustrated in fig. 3.3.



SUBMERGED DROP SPILLWAY

FIGURE 3.3

H_2 = submergence = difference in elevation between tail-water and crest of weir in ft

H_1 = upstream head on weir with negligible velocity of approach = specific energy of flow at the weir where velocity of approach is significant

From the definition of H_1

$$H_1 = H + (v_a^2 \div 2g) \qquad 3.14$$

Then with C = 3.1, equation 3.1 (page 3.1) becomes

$$Q_f = 3.1\ LH_1^{3/2} \qquad 3.15$$

and

$$q_f = 3.1\ H_1^{3/2} = Q_f \div L \qquad 3.16$$

The substitution of $H_1 = H + (v_a^2 \div 2g)$ from equation 3.14 (page 3.16) into equation 3.2 (page 3.7) gives

$$H_1 = h - f \qquad 3.17$$

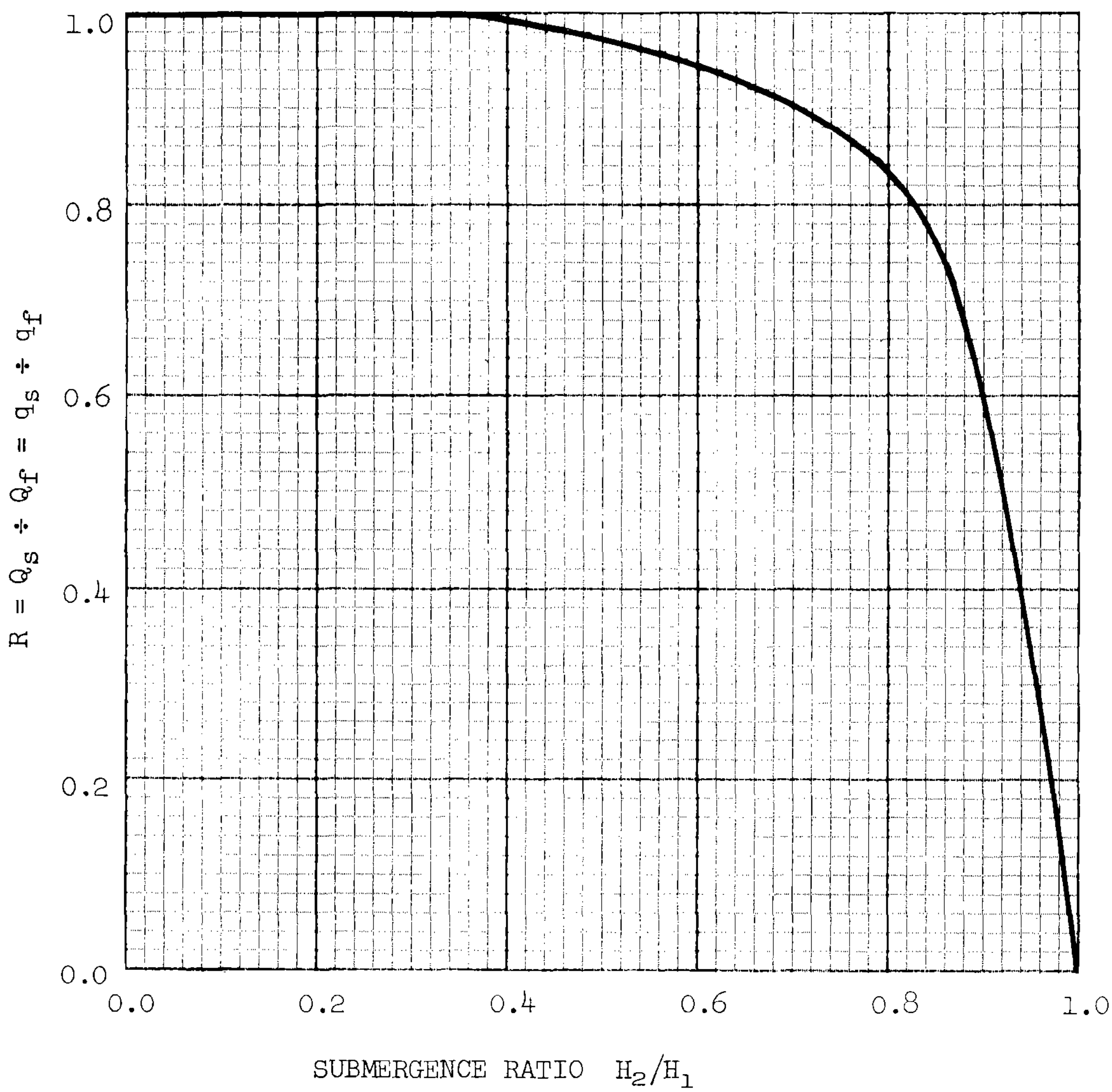


FIGURE 3.4

In a consideration of the effect of submergence, one must first recognize those situations where the effect is apt to exist. Reasonably accurate tailwater elevations will be dependent upon water surface profiles, for various discharges, computed upstream from control points where the stage-discharge relationship is known. See Engineering Handbook, Section 5 on Hydraulics for methods of computing water surface profiles.

Submergence is apt to exist in situations illustrated by the following spillway locations:

1. At the upper end of a drainage ditch where the spillway is designed for a discharge capacity greater than the average bank-full capacity of the drainage ditch below and where the spillway crest elevation is below average ground or bank elevation so as to provide a definite approach channel to the spillway for low flows.

2. Where the spillway is located in a relatively deep gully or channel just upstream from a highway culvert and earth fill, which require and permit a considerable head on the culvert to discharge the spillway design flood.

3. Where the spillway is just upstream from a retarding reservoir in which the maximum flood stage is above the crest of the upstream spillway. Special attention must be given to the element of time in such a problem; will the spillway above the reservoir be required to pass peak or near peak discharge when the reservoir is at or near peak stage? It is quite possible to have a situation in which discharge from other lateral gullies or waterways (runoff from intervening areas) might put the reservoir at or near peak stage at the same time that the spillway under consideration is required to carry maximum discharge.

4. Those in which the channel below the spillway is so flat in grade, so small in cross-sectional area, or so high in resistance to flow that its stage-discharge curve indicates water surface elevations above the spillway crest.

The above possible submergence situations illustrate that the stage just below the spillway may be the primary result of water that has passed over the spillway or of water from some other source. Where the primary source of water producing submergence is above the spillway, remember that the water must first pass through the spillway before it can produce submergence.

Graphs of stage or water-surface elevation just below the spillway, as a function of discharge through the spillway, are valuable and often necessary tools in the solution of submergence problems.

<u>Examples for Submerged Flow</u>. The design of a submerged weir can be accomplished most easily by following a systematic procedure such as outlined and illustrated below. In design, the problem usually resolves itself into one of selecting a certain set of companion values for h and L such that the weir will pass the required discharge rate with a predetermined safe freeboard while operating under tailwater conditions that fix the tailwater elevation (and hence the submergence of H_2) for the design discharge. It is wise to select a somewhat higher freeboard where submergence is of consequence, because of the possible inaccuracies and uncertainties that exist in the computation of the submergence effect.

<u>Example 3.7</u>

Given:
1. $Q = 480$ cfs = required discharge capacity
2. $H_2 = 2.46$ ft = submergence for $Q = 480$ cfs
3. $f = 0.75$ ft = selected freeboard
4. $C = 3.1$ = discharge coefficient

Find: Practical combinations of h and L for a weir that will carry the required peak discharge rate with the associated submergence and the chosen freeboard.

Solution: Obviously, H_1 must exceed H_2 for any discharge to take place. The procedure then becomes a matter of selecting values of h such that H_1 is greater than H_2 and finding companion values of L as indicated in the tabulation below.

Column 1 lists the assumed values of h.

Column 2 gives H_1 as computed from equation 3.17 (page 3.17) for each assumed value of h.

Column 3 gives the values of $H_1^{3/2}$, which can be read directly from table 38, page 103 of the third edition of King's "Handbook of Hydraulics," or they can easily be computed by slide rule.

Column 4 gives the solution of equation 3.16 (page 3.17)

Column 5 gives the ratio $(H_2 \div H_1)$ and is found by dividing the given submergence H_2 (in this case = 2.46) by the values of H_1 given in column 2.

Column 6 lists the values of R that are taken from fig. 3.4 (page 3.17) for each value of the ratio $(H_2 \div H_1)$ given in column 5.

Column 7 gives the solution of equation 3.13 (page 3.15) for values of R and q_f given in columns 6 and 4.

Column 8 lists the results of dividing the total required discharge capacity, Q (in this case = 480 cfs) by the submerged discharge per foot of weir, q_s, from column 7.

Column 9 is merely the result of rounding off the values in column 8 to practical values. It is not practical to detail weir lengths to tenths or any other fraction of a foot.

1	2	3	4	5	6	7	8	9
h	$H_1 = h - f$	$H_1^{3/2}$	$q_f = 3.1\ H_1^{3/2}$	$\frac{H_2}{H_1}$	R	$q_s = Rq_f$	$L = \frac{Q}{q_s}$	L
3.50	2.75	4.56	14.1	0.89	0.63	8.9	53.9	54
4.00	3.25	5.86	18.2	0.76	0.87	15.8	30.4	31
4.50	3.75	7.26	22.5	0.66	0.93	20.9	23.0	23
5.00	4.25	8.76	27.2	0.58	0.95	25.8	18.6	19
5.50	4.75	10.35	32.1	0.52	0.97	31.2	15.4	16
6.00	5.25	12.03	37.3	0.47	0.98	36.6	13.1	13

Comment: In the above tabulation, note the increase in efficiency of the weir, as measured by the value of R, as the value of h increases and the value of $(H_2 \div H_1)$ decreases. In other words, for a fixed amount of submergence the effect of submergence is decreased if the depth of the weir is increased.

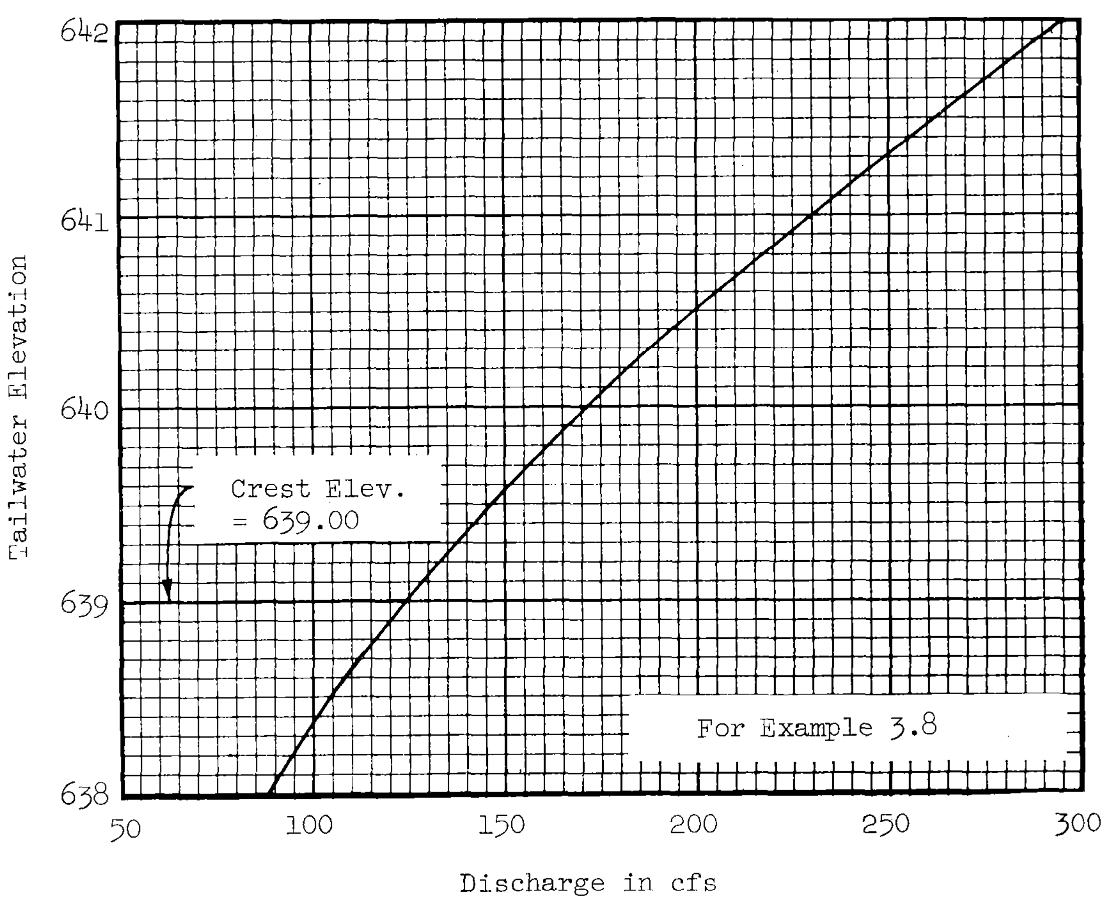
If it is necessary to design a weir with predetermined values of h, f, and H_2, the procedure is illustrated by the computations for any one of the assumed values of h in the previous example.

Should the weir length L be fixed by site conditions or other factors, with predetermined values of f and H_2, the problem becomes one of finding the proper value of h. This problem can be solved by cut-and-try methods, but it is probably easier to prepare a table as illustrated in example 3.7 (page 3.18) for various assumed values of h and plot the relationship between h and L. Then the proper value of h can be picked from this curve for a given value of L, or the value of h can be obtained with sufficient accuracy by interpolation between known companion values of h and L that bracket the required set of conditions.

It may be necessary in some cases to compute the discharge capacity of a given structure (both L and h fixed by existing structure dimensions) that operates under submerged conditions. The solution of such a case is given in the following example.

Example 3.8

Given:
1. Weir dimensions, L = 18 ft; h = 3.50 ft
2. Crest elevation = 639.0
3. Freeboard, f = 0.50 ft
4. Discharge coefficient, C = 3.1
5. Stage-discharge curve for tailwater as given below



Find: Discharge capacity of the weir operating under the specified tailwater conditions and with a freeboard of 0.50 ft as specified.

Solution: The solution depends upon trial and error methods; however, a systematic approach will save time.

First compute the free flow capacity of the weir from equation 3.15 (page 3.16). As pointed out before, $H_1 = H + (v_a{}^2 \div 2g)$.

$$Q_f = 3.1\ LH_1{}^{3/2} = 3.1\ L\ (h - f)^{3/2}$$
$$= 3.1 \cdot 18\ (3.50 - 0.50)^{3/2} = 290\ \text{cfs}$$

Next prepare a table as shown below. A trial value of Q is chosen and the value of H_2 for the assumed trial value of Q is read from the tailwater stage-discharge curve given above. The remaining computations are self-evident. When the trial value of Q equals the submerged discharge, the solution is complete.

Trial Q	H_2	$\frac{H_2}{H_1} = \frac{H_2}{3.00}$	R	$Q_s = RQ_f$ $= R \cdot 290$	Remarks
260	2.47	0.82	0.82	238	high
250	2.32	0.77	0.86	250	O.K.

Layout and Hydraulic Design Criteria. The apron, sidewalls, and wingwalls must perform functions of both structural and hydraulic character. Structurally, they must provide stability against overturning; the sidewalls and wingwalls must retain the embankment and protect it from scour; the apron protects the stream bed against the force of the overfalling water and changes the direction of the flow. In addition to these and related functions, the outlet portion of the drop, including the apron, sidewalls, wingwalls, and attached devices, should be so designed that erosion of the channel bottom and banks just below the spillway will be reduced to a practical minimum.

A considerable amount of research has been conducted to define the proper proportions of the outlet basin and wingwalls, but as yet a satisfactory set of design criteria has not been found.